Water Science and Engineering, Dec. 2008, Vol. 1, No. 4, 47-60
ISSN 1674-2370, *http://kkb.hhu.edu.cn*, e-mail: *wse@hhu.edu.cn*

Mode-I fracture and durability of FRP-concrete bonded interfaces

Qiao Pizhong*[1,2], Xu Yingwu[3]

1. *Department of Civil and Environmental Engineering, Washington State University, Pullman, WA 99164-2910, U. S. A.*
2. *Department of Engineering Mechanics, College of Civil Engineering, Hohai University, Nanjing 210098, P. R. China*
3. *Department of Civil Engineering, The University of Akron, Akron, OH 44325-3905, U. S. A.*

Abstract: In this study, a work-of-fracture method using a three-point bend beam (3PBB) specimen, which is commonly used to determine the fracture energy of concrete, was adapted to evaluate the mode-I fracture and durability of fiber-reinforced polymer (FRP) composite-concrete bonded interfaces. Interface fracture properties were evaluated with established data reduction procedures. The proposed test method is primarily for use in evaluating the effects of freeze-thaw (F-T) and wet-dry (W-D) cycles that are the accelerated aging protocols on the mode-I fracture of carbon FRP-concrete bonded interfaces. The results of the mode-I fracture tests of F-T and W-D cycle-conditioned specimens show that both the critical load and fracture energy decrease as the number of cycles increases, and their degradation pattern has a nearly linear relationship with the number of cycles. However, compared with the effect of the F-T cycles, the critical load and fracture energy degrade at a slower rate with W-D cycles, which suggests that F-T cyclic conditioning causes more deterioration of carbon fiber-reinforced polymer (CFRP)-concrete bonded interface. After 50 and 100 conditioning cycles, scaling of concrete was observed in all the specimens subjected to F-T cycles, but not in those subjected to W-D cycles. The examination of interface fracture surfaces along the bonded interfaces with varying numbers of F-T and W-D conditioning cycles shows that (1) cohesive failure of CFRP composites is not observed in all fractured surfaces; (2) for the control specimens that have not been exposed to any conditioning cycles, the majority of interface failure is a result of cohesive fracture of concrete (peeling of concrete from the concrete substrate), which means that the cracks mostly propagate within the concrete; and (3) as the number of F-T or W-D conditioning cycles increases, adhesive failure along the interface begins to emerge and gradually increases. It is thus concluded that the fracture properties (i.e., the critical load and fracture energy) of the bonded interface are controlled primarily by the concrete cohesive fracture before conditioning and by the adhesive interface fracture after many cycles of F-T or W-D conditioning. As demonstrated in this study, a test method using 3PBB specimens combined with a fictitious crack model and experimental conditioning protocols for durability can be used as an effective qualification method to test new hybrid material interface bonds and to evaluate durability-related effects on the interfaces.

Key words: *repair and strengthening of concrete structures*; *FRP composites*; *FRP-concrete bonded interface*; *mode-I fracture*; *durability*; *freeze-thaw*; *wet-dry*; *interface energy*

DOI: 10.3882/j.issn.1674-2370.2008.04.005

1 Introduction

Increasingly, FRP composites are used for the renewal of civil infrastructure, but there are still concerns about their durability, particularly because the structures of interest are primarily load-bearing and expected to be in service for long periods of time, 30 years or more, without

This study was partially supported by the National Science Foundation (Grant No. CMS-0002829).
*Corresponding author (e-mail: *qiao@wsu.edu*; *qiao@hhu.edu.cn*)
Received Nov. 27, 2008; accepted Dec. 15, 2008

undergoing significant inspection or maintenance.

FRP composites do provide options for rehabilitation, repair and renewal that are not possible with conventional materials. However, newly developed hybrid FRP-concrete structures have not been fully characterized, and the lack of data on performance and durability of FRP-concrete interfaces may deter their widespread application in civil infrastructure. Both moisture and temperature have pronounced effects on properties of the polymer matrix. Therefore, FRP-concrete interface bonds may be degraded or may deteriorate due to environmental exposure. There is a pressing need to investigate the effects of the environment (e.g., temperature, moisture, and chemical conditions) on the long-term durability (with increasing F-T and W-D cycles and accelerated aging) of FRP-concrete bonded interfaces, and to establish a performance database for such a new hybrid material.

Thermal effects in extremely hot and cold climates may influence the properties of the bond between concrete and FRP composites, which is critical in the application of this technology. Polymer resins and adhesives soften over a temperature range, which causes an increase in viscoelastic response, a consequent reduction in elastic mechanical performance levels, and in a number of cases, an increased susceptibility to moisture absorption. Sub-zero temperature exposure can result in matrix hardening, matrix micro-cracking, and fiber-matrix bond degradation. The coefficients of thermal expansion of adhesives can be orders of magnitude different from those of bulk resins and/or composites; hence, thermal gradients/exposure can cause premature de-bonding along the adhesively bonded FRP-concrete interfaces. The effects of cold environments, especially those related to F-T cycling, are of some concern since it is feasible that, in addition to the changes in material response due to cold temperatures, the absorption of moisture into the composites followed by subsequent freezing could lead to increased micro-cracking in the polymer matrix and further degradation due to the cyclic process.

Gomez and Casto (1996) tested two commercially available pultruded fiberglass composite materials immersed in 2% NaCl-water solution and subjected to F-T cycling between –17.8℃ and 4.4℃. Flexural strength and other properties were determined in distinct samples that were tested after various numbers of F-T cycles. Significant property losses occurred when compared with those of uncycled samples. Toutanji and El-Korchi (1998) researched the tensile strength of cylindrical cementitious specimens coated with epoxy and wrapped with three kinds of fiber-reinforced plastic in a reference (room temperature/dry) state and with cyclic W-D and F-T exposures. The carbon fiber systems exhibited small degradations due to the exposures. However, the glass fiber composite sustained a 20% loss in strength under W-D cycling conditions and a 5% loss under cyclic F-T exposure conditions. Steckel et al. (1998) reported the results of a comprehensive experimental program involving twelve composite overwrap systems under a variety of exposure conditions. The systems included carbon/epoxy and glass/epoxy materials, and the exposures included dry heat at 60℃ for 1 000 and 3 000 hours and F-T cycling between –18℃ and 38℃. No loss of stiffness, strength, or failure strain was noted, although one glass and one carbon system lost, respectively, about 15% and 23% of their short beam shear stress when subjected to F-T cycling. Kshirsagar et al. (1998) investigated the durability of fiber-reinforced composite wrapping for the rehabilitation of concrete piers with F-T cycles at temperature ranges of –29℃ to 49℃ and a relative humidity of 100%. Approximately 17 F-T cycles were applied, and a reduction of 2.7% in compressive strength was observed. Green et al. (2000) examined the effects of F-T cyclic conditioning on the bond of FRP sheets to concrete beams. Four different products were considered. The beams were subjected to 50, 150 and 300 F-T cycles consisting of 16 hours of freezing at –18℃ and eight hours of thawing at 15℃. The beams were then tested to failure with four-point bending. The results showed that the bond of FRP sheets to concrete beams was not significantly damaged by the F-T cyclic action. Karbahari and Engineer (1996) examined the external strengthening of concrete beams with composite plates. Unidirectional composites (i.e., either carbon or glass fiber embedded in Tonen or Epon epoxy resins) were wet-impregnated on the surface of concrete beams. The results indicated that although the environmental durability was influenced by the fiber type, the environmental degradation of the flexural strength of the strengthened beams was much higher for the low T_g (glass transition temperature) Tonen resin system than for the Epon system. Katz et al. (1999) investigated the bond properties of FRP reinforcing bars (rebars) at temperatures ranging from a room temperature of about 20℃ to high temperatures of up to 250℃. Test results showed a reduction of between 80% and 90% in the bond strength as the temperature increased from 20℃ to 250℃. Greater sensitivity to high temperatures was seen in FRP rebars, in which the bond relied mainly on the polymer treatment at the surface of the rod. Miyano et al. (1999) researched the time and temperature dependence of static, creep and fatigue behavior for FRP adhesive joints. An adhesively bonded joint was examined experimentally at various temperatures, and it was concluded that the static failure load decreased with increasing temperature. Ferrier and Hamelin (1999) investigated the durability of external carbon epoxy reinforcement of concrete with three kinds of epoxy polymers. The static tension strength was degraded by elevated temperature, but using the polymer with the highest T_g minimized the decrease of mechanical response.

FRP composites are increasingly accepted for use in the seismic retrofitting and strengthening of columns. A number of studies conducted on flat coupon specimens of commercial systems have shown the degradation of these materials when exposed to water or solution. Hence, there is a significant concern related to the life expectancy of the systems when columns are in a floodplain or in high-moisture environments. Beaudoin et al. (1998) investigated the W-D cyclic action on the bond between composite materials and reinforced concrete beams. Reinforced concrete beams externally strengthened on the tension face with layers of carbon/epoxy strips were exposed to cycles of immersion in fresh water at 21℃ for five days followed by two days of drying at 27℃. Flexural tests were conducted at the beginning and after 13 and 26 cycles. Substantial reductions in response were noted for the wet laid-up system with failure in the form of peeling, whereas the prefabricated system showed almost no effect. In cases of premature failure, the failure tended to occur in the composites and concrete rather than along the concrete-composite interface.

A general conclusion drawn from the literature is that the properties of adhesively bonded FRP-concrete products are greatly influenced by exposure conditions such as temperature, moisture, and F-T and W-D cycles. The effects of temperature on mode-I fracture energy of FRP-concrete bonded interfaces have been studied by the authors (Qiao and Xu 2005), and a temperature range from –34.4℃ (–30℉) to 71.1℃ (160℉) has been applied to the fracture tests at increments of about 16.67℃ (30℉). The test results indicated that thermal effects on the properties (e.g., interface fracture energy, critical load, and fractured surface) of adhesively bonded composite-concrete interfaces were pronounced. In this study, a work-of-fracture method using a 3PBB specimen, which is commonly used to determine the fracture energy of concrete (Hillerborg 1985; RILEM Technical Committee 50-FMC 1985), was introduced to evaluate the mode-I fracture and durability of FRP-concrete bonded interfaces. In particular, the proposed test method was implemented to evaluate the effects of F-T and W-D cycles on the mode-I fracture behavior of FRP-concrete bonded interfaces.

2 Determination of mode-I interface fracture energy

In this study, a fictitious crack model was adapted for fracture tests of FRP-concrete bonded interfaces (Xu 2003; Qiao and Xu 2004; Qiao and Chen 2008). The main ingredient of the model is the softening curve, which can describe the fracture process in full. The fracture energy G_F is one of the necessary parameters for determining the softening curve (Qiao and Chen 2008).

A work-of-fracture method for determining the fracture energy of concrete, based on the fictitious crack model, was proposed by Hillerborg (1985) and the RILEM Technical Committee 50-FMC (1985). The method was modified to evaluate the fracture energy of FRP-concrete bonded interfaces (Figure 1) (Qiao and Xu 2004). Because the FRP composite layer being bonded to concrete beams is relatively thin compared to the entire specimen dimensions and the whole specimen is almost symmetrical, mode-I fracture at the bonded interface between the FRP composite and concrete can be produced by this type of specimen.

According to the fictitious crack model (Hillerborg 1985), the fracture energy is the energy required to produce a unit area of crack (full broken). If a specimen is statically fractured and the work (W_F) applied to fracture the interface is measured, the approximate interface fracture energy can be computed as

$$G_F = \frac{W_F}{t(b-a)} \tag{1}$$

where t is the specimen thickness, b is the height of the beam, a is the initial notch (crack) length, and $b-a$ is the initial ligament length of the interface (Figure 1).

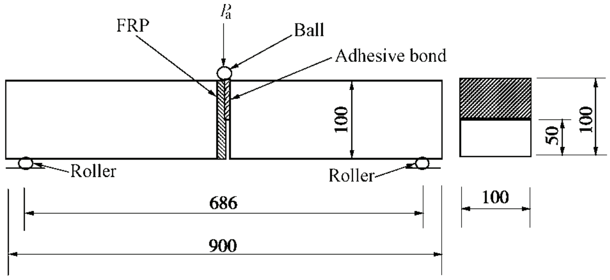


Figure 1 Modified FRP-concrete 3PBB specimens (Unit: mm)

In order to measure the work supply statically, servo-controlled testing is required to measure the complete load-displacement curve. In the test setup of three-point bending notched beams, the weight of the beam specimen contributes to the overall loading of the system. To eliminate such an effect in this study, a special method called weight compensation (Elices et al. 1992; Xu 2003) was adopted (Figure 2).

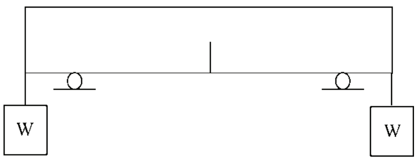


Figure 2 Weight compensation device for three-point bend test

Although it is theoretically possible to compensate for the weight of the beam specimen exactly, in practice it is better to overcompensate so that the dead forces produce a negative bending moment that results in a certain positive constant load P_0' at the end of the test (Figure 3). The work of fracture is the area determined by the load-displacement curve and the line $A'T$ drawn parallel to the displacement axis through the last point T of the recorded curve (Figure 3), where the specimen is assumed to be fully broken. But in the bending test, a complete failure is approached asymptotically, which means that the point T is infinitely far away. Thus, the test usually has to be stopped at point B before all the energy dissipates. We can draw a line AB parallel to the displacement axis, and then there is a small distance P_B' (a load difference) above the true complete failure asymptote. The area enclosed in AMB can be calculated directly, and it is written as W_{m}. The area $A'ABTB'A'$ can also be computed using the methods proposed by Petersson (1981), and approximated as $2P_B'u_B'$, where $u_B' = u_B - u_{A'} \approx u_B - u_A$ and $P_B' = P_B - P_0'$. In the formulas, P_B is the load at point P; and u_B, u_A, and $u_{A'}$ are, respectively, the displacements at points B, A, and A'. Therefore, the total work of fracture (Figure 3) is approximated as

$$W_{\mathrm{F}} = W_{\mathrm{m}} + 2P_B'u_B' \tag{2}$$

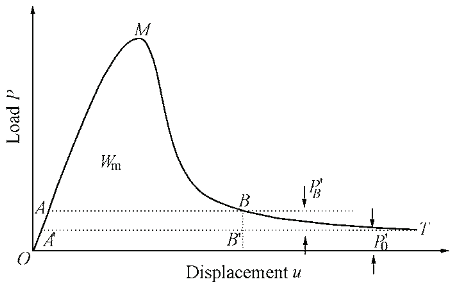


Figure 3 General load-displacement curve for overcompensated three-point bending test

Petersson (1981) also proved that after reaching the peak load P_M or P_{max}, the load-displacement curve behaves asymptotically as u^{-2} for a large value of u, i.e.,

$$P - P_0' = \frac{C}{(u - u_A)^2} \tag{3}$$

where C is a constant. Eq. (3) is satisfied by the last recorded point B, so that

$$P_B - P_0' = P_B' = \frac{C}{(u_B - u_A)^2} \tag{4}$$

After substituting Eq. (4) into Eq. (3), we get

$$P - P_B = C\left[\frac{1}{(u - u_A)^2} - \frac{1}{(u_B - u_A)^2}\right] \tag{5}$$

Therefore, if we plot $P - P_B$ versus $\frac{1}{(u - u_A)^2} - \frac{1}{(u_B - u_A)^2}$, we can determine the constant C in Eq. (3) with the least square fitting of a straight line through the origin. Once the constant C has been determined, Eq. (2) can be rewritten as

$$W_F = W_m + 2\frac{C}{u_B - u_A} \tag{6}$$

From Eq. (4), we can also obtain

$$P_0' = P_B - \frac{C}{(u_B - u_A)^2} \tag{7}$$

Then, the actual critical load can be determined as

$$P_{cr} = P_{max} - P_0' = P_{max} - P_B + \frac{C}{(u_B - u_A)^2} \tag{8}$$

By substituting Eq. (6) into Eq. (1), the fracture energy (G_F) can be computed. In this study, the above data reduction procedures were used to evaluate the fracture energy of FRP-concrete bonded interfaces using a 3PBB specimen (Figure 1).

3 Experimental evaluation

3.1 Materials

Concrete beams with dimensions of 100 mm × 100 mm × 450 mm were made with a mix of cement, sand and aggregate, in respective weight proportions of $1.00:2.17:2.90$. The maximum size of aggregates used was 18 mm, and the water/cement ratio was 0.57. All the beams were removed from the molds 24 hours after casting and then were moisture-cured at 22.2℃ (72℉) for 28 days before they were bonded with FRP composites and subjected to the three-point bending tests.

Carbon fiber fabrics were impregnated with epoxy resins to form the CFRP composite substrates, which were later bonded to the casted concrete beam. The carbon fiber fabric was Toray T700 12k tow sheet with a thickness of 0.165 1 mm/ply (0.006 5 in/ply). An off-the-shelf epoxy resin (Epoxy System 2000), which consisted of two-part mixing components of resin and hardener, was used.

3.2 Specimen preparation and conditioning protocol for F-T and W-D cycles

There is no standard practice for the testing of FRP-concrete interfaces subjected to F-T conditions and immersed in calcium chloride solutions. For this reason, ASTM C 672/C 672M-03(ASTM Committee C09 2003) was reviewed and modified appropriately in order to formulate the present experimental conditioning protocol.

To fabricate the 3PBB specimens for interface durability characterization, the end cross sections of two equal-sized concrete beams were first sand-blasted and then bonded with composite fabric to form a CFRP-concrete interface (Figure 4). The curing time of the bonded interfaces was seven days under room temperature. An initial central notch, which was half of the beam height, was created along the interfaces using a cellophane insert. A total of 24 CFRP-concrete 3PBB specimens, shown in Figure 4, were fabricated and divided into four groups (six samples in each group), 50 F-T cycles, 100 F-T cycles, 50 W-D cycles, and 100 W-D cycles, in order to evaluate the effects of F-T and W-D cycles on fracture energy.

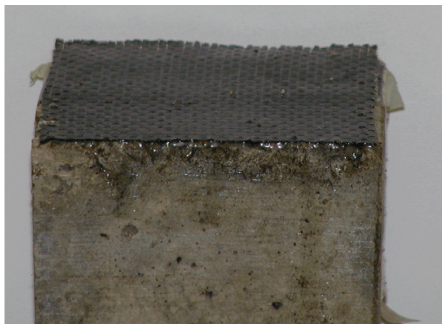


Figure 4 Forming of CFRP-concrete bonded interface (half of the 3PBB specimen)

For the F-T effect, the interface samples were put into containers after the curing of the bonded interfaces, and the interfaces were immersed in a solution of calcium chloride and water that had a mass concentration of 0.04 g/mL. About 50 mm of the half portion of the 3PBB specimen containing the CFRP-concrete bonded interface as illustrated in Figure 4 was immersed in the solution with the interface facing down and toward the bottom of the container. Then, the interface samples, along with the container, were conditioned in a freezer at about –18℃ (0℉) for 16 hours. At the end of the 16-hour freezing, the interface samples were removed from the freezer and thawed at a temperature of 22℃ (72℉) for eight hours. The 16-hour freezing and eight-hour thawing (total of about one day) constituted one cycle, and the cycles were repeated for 50 and 100 days to conduct 50 and 100 F-T conditioning cycles. At the end of 50 and 100 cycles, the conditioned interface samples were bonded with other concrete beams to form the 3PBB specimens. The F-T cycle-conditioned 3PBB specimens were then cured for seven days before they were tested with 3-point bending.

For the W-D effect, the interface samples were placed in the containers after the curing of the bonded interfaces as shown in Figure 4, and the interfaces were immersed to about 50 mm depth in a solution of calcium chloride and water which had a mass concentration of 0.04 g/mL. After 16 hours of wetting in the containers, the interface samples were removed and air dried. Both dry and wet conditionings were conducted at room temperature (22℃, 72℉). One cycle of W-D conditioning consisted of 16 hours of wetting and eight hours of drying, and was repeated for 50 and 100 days in order to conduct 50 and 100 W-D conditioning cycles. As in the case of the F-T samples, at the end of 50 and 100 cycles, another concrete beam was bonded to the conditioned interface sample to form the 3PBB specimen. The W-D cycle-conditioned 3PBB specimens were tested after seven days of curing.

Six CFRP-concrete 3PBB specimens without any conditioning were also fabricated. To fabricate the specimens, the end cross sections of two equal-sized concrete beams were first sand-blasted and then bonded with carbon composite fabrics to form CFRP-concrete interfaces. The curing time of the bonded interfaces was seven days at room temperature. An initial central notch, which was half the length of the beam height, was created along the interfaces using a cellophane insert. These six unconditioned samples were considered benchmark specimens for comparison with the F-T and W-D conditioned samples.

3.3 Mode- I fracture testing

The test configuration of a CFRP-concrete bonded 3PBB specimen is shown in Figure 5. A servo-hydraulic MTS machine was used to conduct the experiment, and the central applied load and load point displacement were recorded A clip gage was used to accurately measure the load point displacement. A loading rate of 0.01 mm/s was employed in the tests. Both the load and load point displacement for each sample were measured.

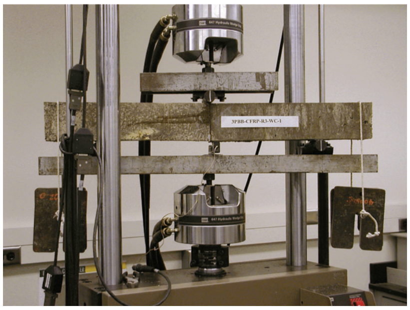


Figure 5 Test configuration of 3PBB specimen for mode-I fracture testing of CFRP-concrete bonded interface

4 Results and discussion

The applied load vs. load point displacement curves of the interfaces with varying numbers of F-T and W-D cycles (0, 50, and 100) are shown in Figures 6 and 7. The area covered under the load-displacement curve represents the work necessary to fully break apart the interface. Based on the data reduction technique introduced in Section 2, the interface fracture energy (G_F) and critical load (P_{cr}) with varying numbers of F-T and W-D conditioning cycles were obtained and are given in Tables 1 and 2. The effects of F-T and W-D cycles on critical load are shown in Figure 8: the critical load significantly decreases as the number of cycles increases, and this trend almost follows a linear proportional pattern. The influences of F-T and W-D cycles on the fracture energy of CFRP-concrete bonded interfaces are illustrated in Figure 9, and a similar trend in fracture energy with critical loads is observed. However, compared with the effect of F-T cycles, the critical load and fracture energy decrease at a relatively slower rate with W-D cycles, which indicates that the F-T cycles cause more degradation of the properties of FRP-concrete bonded interfaces.

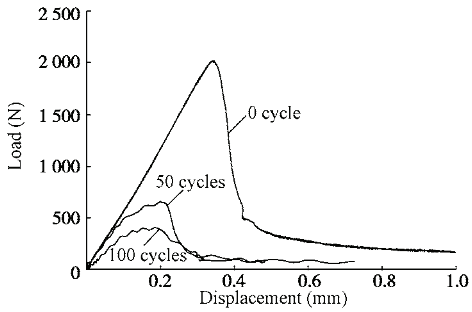


Figure 6 Load-displacement curves with varying numbers of F-T cycles

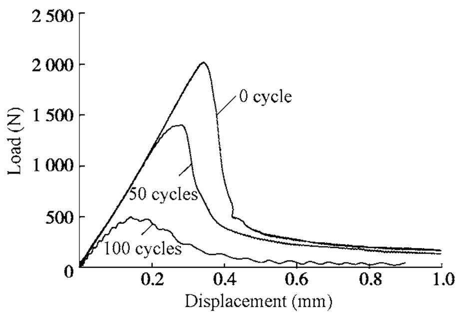


Figure 7 Load-displacement curves with varying numbers of W-D cycles

Table 1 Effects of F-T cycles on G_F and P_{cr} of CFRP-concrete bonded interfaces

specimen	G_F (N/m)			P_{cr} (N)		
	$n_{F\text{-}T} = 0$	$n_{F\text{-}T} = 50$	$n_{F\text{-}T} = 100$	$n_{F\text{-}T} = 0$	$n_{F\text{-}T} = 50$	$n_{F\text{-}T} = 100$
1	112.4	67.7	20.4	1 764.3	897.5	260.0
2	129.7	64.8	21.7	2 180.1	915.0	505.4
3	110.3	66.7	7.8	1 731.5	1 196.3	302.5
4	98.7	67.5	15.4	1 673.9	992.7	346.4
5	106.8	56.1	9.8	1 942.1	1 095.6	308.1
6	120.9	60.9	10.5	1 895.5	809.9	417.2
COV	0.096	0.072	0.409	0.099	0.144	0.252

Note: $n_{F\text{-}T}$ is the number of F-T cycles, and *COV* is the coefficient of variation.

Table 2 Effects of W-D cycles on G_F and P_{cr} of CFRP-concrete bonded interfaces

specimen	G_F (N/m)			P_{cr} (N)		
	$n_{W\text{-}D} = 0$	$n_{W\text{-}D} = 50$	$n_{W\text{-}D} = 100$	$n_{W\text{-}D} = 0$	$n_{W\text{-}D} = 50$	$n_{W\text{-}D} = 100$
1	112.4	65.0	34.9	1 764.3	1 349.5	523.1
2	129.7	68.3	41.0	2 180.1	1 097.5	561.2
3	110.3	76.3	34.9	1 731.5	1 330.5	971.8
4	98.7	85.6	27.7	1 673.9	1 644.4	485.7
5	106.8	84.1	34.9	1 942.1	1 573.0	343.8
6	120.9	85.2	22.1	1 895.5	1 580.5	303.4
COV	0.096	0.117	0.204	0.099	0.145	0.449

Note: $n_{W\text{-}D}$ is the number of W-D cycles, and *COV* is the coefficient of variation.

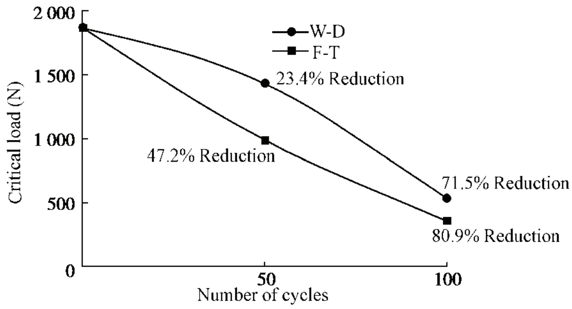


Figure 8 Comparison of effects of W-D and F-T cycles on critical load

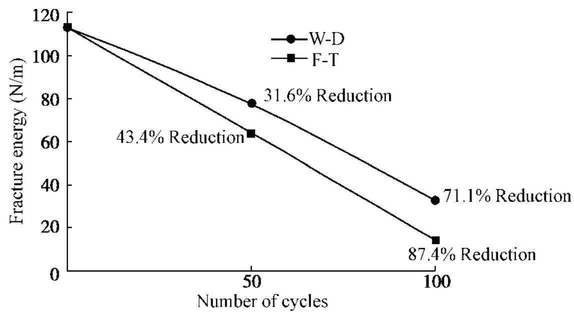
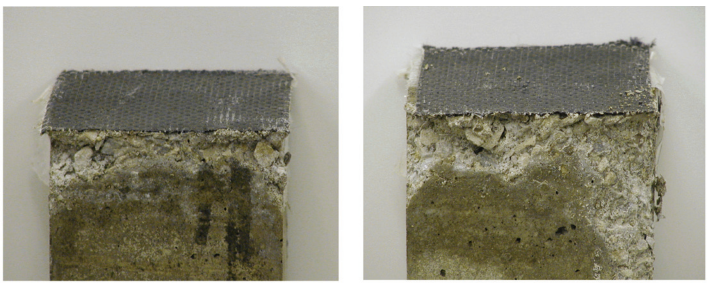


Figure 9 Comparison of effects of W-D and F-T cycles on interface fracture energy

At the end of 50 and 100 F-T or W-D cycles, the concrete portions near the CFRP-concrete interfaces were examined. Scaling (the flaking or peeling-off of surface mortar or concrete, which is usually caused by freezing and thawing of concrete in damp or water-saturated environments) occurred in all the specimens subjected to F-T cycles near the bonded interfaces (Figures 10(a) and 10(b) for the effects of 50 and 100 F-T cycles, respectively). Besides the degradation of the bonded interface, scaling of concrete is one of the most insidious types of deterioration. The scaling of concrete was not observed in the specimens undergoing W-D conditioning cycles.

The surfaces of interface fractures along the bonding lines with varying numbers of F-T and W-D cycles were also examined, and the percentages of adhesive failure along the interface bond and cohesive failures of concrete are reported in Tables 3 and 4. The cohesive failure of CFRP composite (i.e., the failure within the CFRP composite layer) was not observed in the fractured surfaces and therefore is not reported in Tables 3 and 4. Tables 3 and 4 reveal that, for the CFRP-concrete specimens that were not exposed to environmental cycles, the majority of interface failure is the result of the cohesive fracture of concrete (peeling of concrete from the concrete substrate), as concluded in another study by the authors (Qiao and Xu 2004). This means that the cracks mostly propagate within the concrete (Figure 11(a)). Cohesive interface fracture of this type of non-conditioned specimen was simulated by Qiao and Chen (2008). As the number of F-T and W-D cycles increases, the adhesive failure along the interface begins to appear, gradually increases, and eventually becomes a dominant failure mode (Figures 11(b) through 11(e)). The fractured surfaces shown in Figures 11(a) through 11(e) illustrate the gradual transformation of the failure mode from the cohesive failure of concrete to adhesive failure of the interface as the number of F-T and W-D cycles increases, manifesting the decay and degradation of the CFRP-concrete bonded interface under accelerated environmental conditioning, i.e., the repeated F-T and W-D cycling in this study. It is therefore concluded that the fracture properties (e.g., critical load, fracture energy, and percentage of fracture surface) of the bonded interface are controlled by the concrete cohesive fracture in the absence of environmental cycles, and by the adhesive fracture with an increased number of F-T or W-D cycles (such as 100).



(a) 50 F-T cycles (b) 100 F-T cycles

Figure 10 Scaling of concrete

Table 3 Failure percentage of fractured surface for CFRP-concrete bonded interface with F-T cycles

specimen	F_{ad} (%)			F_{con} (%)		
	$n_{F\text{-}T}=0$	$n_{F\text{-}T}=50$	$n_{F\text{-}T}=100$	$n_{F\text{-}T}=0$	$n_{F\text{-}T}=50$	$n_{F\text{-}T}=100$
1	25	70	95	75	30	5
2	16	80	90	84	20	10
3	20	60	95	80	40	5
4	10	40	100	90	60	0
5	5	70	100	95	30	0
6	15	50	95	85	50	5

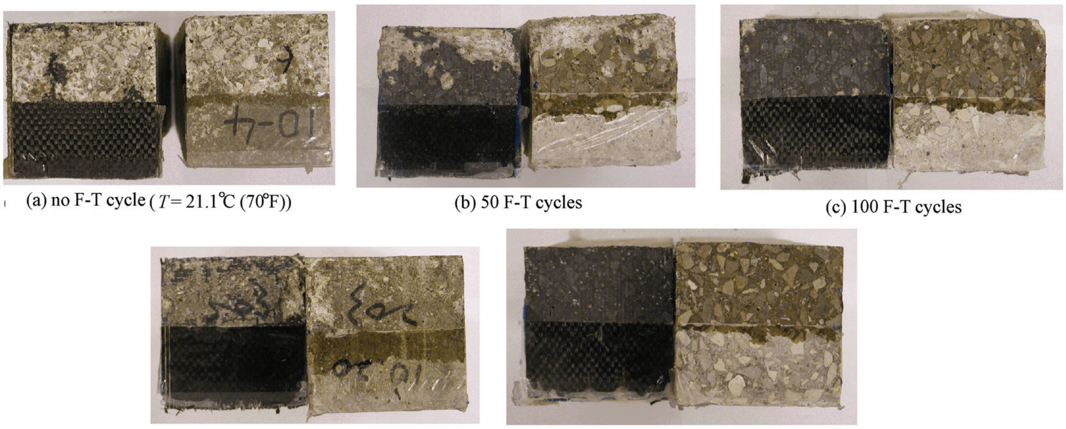
Note: $n_{F\text{-}T}$ is the number of F-T cycles, F_{ad} is the percentage of interface adhesive failure, and F_{con} is the percentage of concrete cohesive failure.

Table 4 Failure percentage of fractured surface for CFRP-concrete bonded interface with W-D cycles

specimen	F_{ad} (%)	F_{con} (%)

	$n_{W\text{-}D}$ = 0	$n_{W\text{-}D}$ = 50	$n_{W\text{-}D}$ = 100	$n_{W\text{-}D}$ = 0	$n_{W\text{-}D}$ = 50	$n_{W\text{-}D}$ = 100
1	25	40	100	75	60	0
2	16	20	60	84	80	40
3	20	25	70	80	75	30
4	10	10	100	90	90	0
5	5	20	100	95	80	0
6	15	30	90	85	70	10

Note: $n_{W\text{-}D}$ is the number of W-D cycles, F_{ad} is the percentage of interface adhesive failure, and F_{con} is the percentage of concrete cohesive failure.



(a) no F-T cycle (T = 21.1℃ (70℉)) (b) 50 F-T cycles (c) 100 F-T cycles

(d) 50 W-D cycles (e) 100 W-D cycles

Figure 11 Fractured surfaces of CFRP-concrete interface

5 Conclusions

In this study, a work-of-fracture method to determine the fracture energy of concrete proposed by Hillerborg (1985) and the RILEM Technical Committee 50-FMC (1985) was adapted to evaluate the fracture energy of FRP-concrete bonded interfaces. The established data reduction procedures were used to evaluate the critical fracture load and fracture energy of FRP-concrete bonded interfaces using 3PBB tests, which produce interface fracture under mode-I loading. The test method was used to characterize the effects of F-T and W-D conditioning cycles on the fracture behavior of FRP-concrete bonded interfaces, and the interface fracture energy was evaluated based on a fictitious crack model.

In this study, carbon fiber fabric was used, and a common epoxy resin was applied to bond the carbon fabric to concrete so that a CFRP-concrete bonded interface was fabricated. Mode-I fracture tests of the 3PBB specimens for CFRP-concrete bonded interfaces were performed to determine the applied load and load point displacement relationship, from which the interface fracture energy was computed. The test results of F-T and W-D effects indicate that the critical load and fracture energy both decrease as the number of F-T and W-D cycles increases, following an inverse, almost proportionally linear pattern. However, compared to the effect of F-T cycles, the critical load and fracture energy decreased at a slower rate with W-D cycles, which suggests that the F-T cycles cause more degradation of the properties of

CFRP-concrete bonded interfaces. After 50 and 100 cycles, scaling of the concrete was found in all specimens that experienced F-T cycles but not in the specimens that experienced W-D cycles. The interface fracture surfaces along the bonding lines with varying numbers of F-T and W-D cycles were also examined. Only the concrete cohesive failure and interface adhesive failure occur at the fractured interfaces, and the cohesive failure of CFRP composites is not observed in all the fractured surfaces. The results reveal that, for specimens that are not exposed to F-T or W-D cycles, the majority of interface failure is the result of cohesive fracturing of concrete (peeling of concrete from the substrate near the interface bonding line), which means that the cracks mostly propagate within the concrete. As the number of F-T and W-D cycles increases, the adhesive failure along the interface begins to appear, gradually increases, and eventually prevails. It is therefore concluded that the fracture properties (critical load and fracture energy) of the bonded interface are controlled by the concrete cohesive fracture in the absence of environmental conditioning cycles and finally by the adhesive fracture with high numbers of F-T or W-D cycles, and both the F-T and W-D conditionings induce interface degradation and reduce fracture properties. In particular, the F-T conditioning cycles cause more degradation of the interface and thus accelerate the aging of the interface.

As demonstrated in this study, the test method using 3PBB specimens for mode-I interface fracture characterization and the environmental conditioning protocols using the F-T and W-D cycles can be effectively used to evaluate and rank new hybrid material interface bonds and to study durability-related effects on the interface.

Acknowledgement

The first author would like to express his gratitude to the Changjiang (Cheung Kong) Scholar Award from the Ministry of Education of the People’s Republic of China.

References

American Society for Testing and Materials (ASTM) Committee C09. 2003. *Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals (ASTM C 672/C 672M-03)*. West Conshohocken: ASTM International.

Beaudoin, Y., Labossiere, P., and Neale, K. W. 1998. Wet-dry action on the bond between composite materials and reinforced concrete. Benmokrane, B., and Rahman, H., eds., *Proceedings of the 1st Conference on Durability of Fiber Reinforced Polymer (FRP) Composites for Construction (CDCC 1998)*, 537-546. Sherbrooke: University of Sherbrooke.

Elices, M., Guinea, G. V., and Planas, J. 1992. Measurement of the fracture energy using three-point bend tests: Part 3: Influence of cutting the P-δ tail. *Materials and Structures*, 25(6), 327-334.

Ferrier, E., and Hamelin, P. 1999. Durability of external carbon epoxy reinforcement of concrete. *Proceedings of the 12th International Conference on Composite Materials (ICCM-12)*. Paris: Woodhead Publishing Ltd.

Gomez, J., and Casto, B. 1996. Freeze/thaw durability of composite materials. *Proceedings of the 1th International Conference on Composites in Infrastructure (ICCI): Fiber Composites in Infrastructure*, 947-955. Tucson: University of Arizona.

Green, M. F., Bisby, L. A., Beaudoin, Y., and Labossiere, P. 2000. Effect of freeze-thaw cycles on the bond durability between fibre reinforced polymer plate reinforcement and concrete. *Canadian Journal of Civil Engineering*, 27(5), 949-959.

Hillerborg, A. 1985. The theoretical basis of a method to determine the fracture energy G_F of concrete. *Materials and Structures*, 18(4), 291-296.

Karbhari, V. M., and Engineer, M. 1996. Effect of environmental exposure on the external strengthening of concrete with composites-short term bond durability. *Journal of Reinforced Plastics and Composites*, 15(12), 1194-1216.

Katz, A., Berman, N., and Bank, L. C. 1999. Effect of high temperature on bond strength of FRP rebars. *Journal of Composites for Construction*, 3(2), 73-81.

Kshirsagar, S., Lopez-Anido, R., and Gupta, R. K. 1998. Durability of fiber reinforced composite wrapping for the rehabilitation of concrete piers. *Proceedings of the 1st Conference on Durability of Fiber Reinforced Polymer (FRP) Composites for Construction (CDCC 1998)*, 117-128. Sherbrooke: University of Sherbrooke.

Miyano, Y., Nakada, M., and Yonemori, T., Sihn, S., and Tsai, S. W. 1999. Time and temperature dependence of static, creep and fatigue behavior for FRP adhesive joints. *Proceedings of the 12th International Conference on Composite Materials (ICCM-12)*. Paris: Woodhead Publishing Ltd.

Petersson, P. E. 1981. *Crack Growth and Development of Fracture Zones in Plain Concrete and Similar Materials, Report TVBM-1006*. Lund: Division of Building Materials, Lund Institute of Technology.

Qiao, P. Z., and Xu, Y. W. 2004. Evaluation of fracture energy of composite-concrete bonded interfaces using three-point bend tests. *Journal of Composites for Construction*, ASCE, 8(4), 352-359.

Qiao, P. Z., and Xu, Y. W. 2005. Thermal effects on the fracture of adhesively bonded composite-concrete interface. *Journal of Advanced Materials*, 37(2), 56-62.

Qiao, P. Z., and Chen, Y. 2008. Cohesive fracture simulation and failure modes of FRP-concrete bonded interfaces. *Theoretical and Applied Fracture Mechanics*, 49(2), 213-225.

RILEM Technical Committee 50-FMC. 1985. Determination of the fracture energy of mortar and concrete by means of three-point bend tests on notched beams. *Materials and Structures*, 18(6), 285-290.

Steckel, G. L., Hawkins, G. F., and Bauer, J. L. 1998. Environmental durability of composites for seismic retrofit of bridge columns. *Proceedings of the 2nd International Conference on Composites in Infrastructure (ICCI): Fiber Composites in Infrastructure*, 2, 460-475. Tucson: University of Arizona.

Toutanji, H., and El-Korchi, T. 1998. Tensile durability performance of cementitious composites externally wrapped with FRP sheets. *Proceedings of the 2nd International Conference on Composites in Infrastructure (ICCI): Fiber Composites in Infrastructure*, 2, 410-422. Tucson: University of Arizona.

Xu, Y. W. 2003. Fracture and Durability of FRP-concrete Interface Bonds. M.S. Thesis. Akron: The University of Akron.